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16. ABSTRACT

Synopsis

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The importance of adequate exploration, testing and analysis is emphasized. The uncertainties and limitations of the application of theoretical soil mechanics principles are pointed out. The use of field permeabilities is proposed for calculating rates of settlement, and a method of measuring in-place permeability is described. Conventional methods of predicting settlement, derived from the theory of consolidation, are not always reliable when applied to fibrous peat. Examples are presented of embankment construction across peat beds, with comparisons of theoretical and observed settlement.

No one method of construction across marsh deposits is suitable or economical for all conditions. After thorough exploration and testing, the stability and settlement can be estimated for different designs by applying the principles of soil mechanics. The choice of design will usually be based on cost comparisons, taking into account all cost factors and considering the adequacy of service to the highway user.

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STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

CALIFORNIA EXPERIENCE IN CONSTRUCTION OF HIGHWAYS ACROSS MARSH DEPOSITS

By

A. W. Root

Supv. Materials and Research Engineer

Presented at the 36th Annual Meeting
of the Highway Research Board
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CALIFORNIA EXPERIENCE IN CONSTRUCTION OF HIGHWAYS ACROSS MARSH DEPOSITS

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Embankments have been constructed across marsh deposits in California by several different methods, such as: by controlled rate of placement and use of berms; by removal of soft compressible material, by stripping or dredging; by displacement of the weak soil by the embankment; and by the installation of vertical sand drains. Examples of each type of design are cited, with plots of observed settlement.

The importance of adequate exploration, testing and analysis is emphasized. The uncertainties and limitations of the application of theoretical soil mechanics principles are pointed out. The use of field permeabilities is proposed for calculating rates of settlement, and a method of measuring in-place permeability is described. Conventional methods of predicting settlement, derived from the theory of consolidation, are not always reliable when applied to fibrous peat. Examples are presented of embankment construction across peat beds, with comparisons of theoretical and observed settlement.

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INTRODUCTION

Construction of highways across marsh deposits always presents an important and difficult engineering problem, usually expensive to solve. As a result of improper design, excessive differential settlement may create a hazard to the highway user and necessitate costly reconstruction or repairs; or, the embankments may be so unstable that failures or slip-outs occur during construction and perhaps throughout the life of the facility. Placing the highway across marsh areas on pile trestles or structures is costly, and is not always a satisfactory solution, as the integrity of the structures may be jeopardized by the effects of settlement unless the piles are designed to resist the loading incurred by negative skin friction resulting from settlement of the marsh deposits.

In designing embankments two basic questions must be answered: (1) will the foundation soil support the required embankment load without displacement or shear failures; and (2) will excessive settlement occur due to consolidation of the marsh soil. Fortunately, by the application of soil mechanics, reasonably accurate solutions of these problems are possible in most cases. Methods for analysing settlement by use of the theory of consolidation have been set forth by Terzaghi[1]. The reliability of the answers depends largely on the thoroughness of the exploration, the quality of undisturbed samples, the use of suitable test procedures, and most important the proper analysis and interpretation of the test data.

Based on the results of such soil tests and analysis, the soil engineer can estimate the permissible loading and the magnitude and rate of settlement for various loading conditions and methods of construction; the design then becomes largely a matter of engineering economics. If the marsh deposits comprise thick layers of weak highly compressible mud or peat, construction is likely to be costly, and the marsh area should be avoided if possible. But too often avoidance of the unstable areas will result in circuitous routing or substandard alignment, leaving no satisfactory alternative but to traverse the marsh area.

Several methods of construction have been used successfully in California, including: (1) controlled rate of placement and use of berms; (2) partial or complete dredging or stripping of the weak compressible soil; (3) displacing the soft material by the weight of the embankment without stripping; (4) use of vertical sand drains.

All of the special treatments are relatively costly, and frequently funds are not available to construct a project if stripping or sand drains are required. On less important routes carrying light traffic the cost of extensive foundation treatment may not be warranted. If stability analyses show that the marsh soil will support the required height of embankment without danger of shear failures, the embankments may be constructed by conventional methods. If the analyses indicate that the factor of safety is dangerously low the embankment should be designed with berms and the rate of placement of the fill material should be carefully controlled. Embankments constructed by this method over thick layers of highly compressible soil will continue to settle over a long period of time. The post-construction settlement can usually be reduced by placing an overload or surcharge, provided the construction schedule is such that the surcharge can be left in place for a sufficient length of time prior to paving. Surcharges must be used with discretion, however, as the weak foundation soil often will not safely support the additional load. Furthermore, the surcharge is worse than useless unless left in place long enough to effect appreciable consolidation, which often requires a long waiting period.

It is important that reliable estimates be made of the magnitude and rate of settlement in order to determine whether the settlement can be tolerated, and for evaluating the probable cost of maintenance, including periodic reconstruction when required. Where interchange structures or bridges might be damaged by settlement of the approach fills, special foundation treatment, such as stripping or sand drains, is almost mandatory in the areas adjacent to the structures.

CONSTRUCTION WITHOUT TREATING THE MARSH SOIL

Low embankments have been constructed across marsh areas without removal or treatment of the weak soil on several highway projects in California. Figure 1 shows plots of observed settlement at two points on a low embankment constructed over a marsh area where the soil consisted of 7 to 12 feet of soft silty clay (described locally as "bay mud"), underlain by stiff sandy clay. On this project sand drains were installed in the embankment areas adjacent to structures, but no special foundation treatment was applied on other portions of the project, where the height of fill did not exceed six or eight feet. Differential settlement after paving has resulted in noticeable distortion of the roadway profile and cross-section.

The easterly approach of the San Francisco-Oakland Bay Bridge is one of the earlier California projects involving embankment construction across thick deposits of very weak, highly compressible soil; here an embankment was constructed across a long section of tidal land where the depth of soft, silty clay (bay mud) was as much as eighty feet, and the height of fill was generally fifteen to twenty-five feet above the mud-line. About twelve feet of the very soft mud was removed by dredging, after which the sand fill was placed hydraulically, in stages. Settlement of the embankment has been recorded during the twenty years which have now elapsed since the fill was placed. Figure 2 is the settlement curve for a reference point at Station 317+50 where the depth of soft material prior to dredging was about seventy feet.

Note that the settlement to date at this point has been about seven feet. During the period from one to ten years after construction, the settlement was proportional to the logarithm of time, and occurred at a diminishing rate thereafter; the slope of the curve indicates that the primary consolidation may now be nearing completion. Figures 3, 4 and 5 are somewhat typical of the laboratory load-consolidation and time-consolidation curves, respectively, for the soft silty clay in this vicinity.

It is of interest to note that, although the settlement of this fill subsequent to paving ranged from 1 ft. to 8 ft. in a length of about 8000 ft., the differential settlement has not seriously affected the riding qualities of the road and has not impaired the service to traffic.

STRIPPING OF THE UNSTABLE SOIL

If the thickness of the weak compressible soil layer is relatively small the most economical and reliable treatment may be to strip and waste the unstable soil. This method was used on a project near Petaluma, California, where a high fill was to be constructed across a marshy area where the soil consisted of a ten-foot layer of very soft compressible soil, underlain by a stiffer and less compressible soil. The soft material was stripped to a depth of eight to ten feet and replaced by granular material from roadway excavation. Figure 6 is a plot of observed settlement of a point set at the bottom of the fill in the stripped area, where the thickness of embankment over the settlement platform was 69 ft. The observed settlement was due primarily to consolidation of a sandy clay layer eleven feet in thickness underlying the soft material which was stripped. The lapse of time between grading and pavement was such that the settlement was virtually completed before the pavement was placed. On other projects, where all compressible soil was stripped, practically no movement was recorded.

On the above project stripping was planned because the soil had such low strength that it would not support the required height of embankment. On other projects, however, compressible soil has been stripped under approach fills at structures in order to prevent excessive settlement which would damage the structure, even though the shear strength of the soil might be sufficient to permit construction of the proposed fill without failures or slipouts of the fill.

If the depth of weak compressible soil is great, it is usually impractical to strip down to firm material. In such cases it may be possible to construct the required embankment by removing only a portion of the weak material, by stripping to a depth which will permit the necessary loading without shear failures. Careful analysis must be made to determine the required depth of stripping, and even though no shear failures occur settlement of the embankment may continue for a long period of time. Partial stripping is seldom recommended if other more positive types of treatment are feasible.

DISPLACEMENT OF MARSH SOIL BY WEIGHT OF THE EMBANKMENT

Displacement of peat by blasting has been practiced extensively in other regions, but very little work of this type has been done in California. The railroads have had considerable experience with displacement of soft mud by end-dumping rock into swamp areas until a more or less stable fill resulted. In most cases no concerted effort was made to displace all of the soft material, and often the completed embankment was subject to periodic shear failures; or, even if actual failures did not occur, the differential settlements over a long period of time were so great that maintenance costs were excessive, and normal service was difficult or impossible to maintain.

Construction of a freeway south of San Francisco, between Sierra Point and Candlestick Point, necessitated the crossing of an arm of the bay where the depth of water was about five feet; the soil in the bay at this location consisted of from forty to eighty feet of soft silty clay or "bay mud" underlain by relatively firm sandy clay or sand. After exploring the area and studying various methods of construction, it was decided to construct a short experimental section of embankment to determine whether sufficient mud could be displaced without dredging to permit construction of a stable embankment. It was found that an embankment of the required height could not be "floated" on the soft mud, but that with proper control of the embankment construction most of the soft material could be displaced during the filling operation(2). The embankment has now been completed across the 2 mile length of open water, and paving of the roadbed is in progress. It was, of course, impossible to displace all of the mud, and considerable settlement is anticipated. Figure 7 shows two cross-sections of the fill, and Figure 8 shows the recorded settlement at the same locations. At Station 58 the elevation of the bottom of the soft mud is about -67, and the thickness of mud remaining under the fill ranged from 7 ft. to 37 ft. at various locations on the section; corresponding values for Station 89 are: bottom of mud elev. -49, and thickness of remaining mud 0 to 25 ft. Embankment was constructed to about elevation +19 at both stations.

VERTICAL SAND DRAINS

Numerous papers have been published describing the use of vertical sand drains in construction of embankments over marsh deposits. The first installation of sand drains by the California Division of Highways in 1934 has been described by O. J. Porter(3). Since that time vertical sand drains have been installed on numerous projects.

As a comprehensive survey and evaluation of sand drain installations throughout the United States is now in progress by others, this paper will not present any detailed analysis of sand drain projects in California. It is the author's opinion, based on study of sand drain projects in California, that when properly used, sand drains are effective in increasing the shear strength of marsh soils during the loading period and reducing settlement subsequent to construction. It is emphasized, however, that vertical sand drains are not a panacea for all foundation troubles, and should not be used indiscriminately.

Success of vertical sand drain treatment depends largely on two factors: (1) the rate of increase in shear strength of the soil must be sufficient to prevent the occurrence of shear failures; (2) the rate of consolidation of the foundation soil must be such that the major portion of the settlement will occur during construction, and subsequent settlement will be within tolerable limits. It should be obvious that sand drains cannot be used successfully if the shear strength of the soil during the loading period will be so low that shear failures are inevitable, or if the soil consolidates at such a slow rate that primary settlement is not completed until years after the embankment is constructed. Yet there are records of projects where sand drains were installed under such conditions, either because of neglecting to make the necessary stability calculations, or because of inaccuracies in computing the time rate of settlement.

There are many uncertainties and indeterminates involved in the analyses of stability and settlement in connection with sand drains, as will be discussed later. These known difficulties, however, strongly emphasize the importance of thorough, meticulous sampling, testing, and analysis before adopting sand drains as a treatment of marsh deposits.

Sand drains were installed at the approaches to the Eureka Slough Bridge in northern California, where embankments up to 25 feet in height were to be constructed over a layer of weak compressible soil. Figure 9 is a plot of the observed settlement at one of the settlement devices, where the embankment was 21 feet in height and the thickness of mud was about 20 feet. A total settlement of 1.5 ft. has been recorded, but only about 0.2 ft. has taken place after paving. No fill failures occurred in the sand drain area.

Recent construction north of the Antioch Bridge in the Delta area of California necessitated construction of embankments over deposits of fibrous peat. Sand drains were installed in two areas, one adjacent to the bridge end, and the other where the new roadbed was located along the toe of an existing levee. At the latter location, where the height of embankment was to be 6 to 8 feet above original ground surface, there was 28 feet of soft fibrous peat and 22 feet of peaty clay underlain by firm silty sand. A typical cross-section of the completed embankment is shown in Figure 10. The contact between the fill and the peat was determined by borings. There was no evidence of shear failures or displacement. Settlement of about 12 feet occurred at Station 80+10, for which the time-settlement curve is shown in Figure 15. Maximum settlement of over 18 feet was recorded in this area.

TIME RATE OF CONSOLIDATION

The strength and load-consolidation characteristics of saturated inorganic clays can be evaluated with considerable confidence. There is, however, some uncertainty in estimating time-consolidation relationships, except in the rare instances where the soil is relatively homogeneous and isotropic. The effect of sand drains on the time rate of settlement is computed by the method proposed by Barron(4). This calculation requires determination of the coefficient of consolidation in both the vertical and horizontal directions. Although horizontal permeability is probably the most important factor in estimating the effect of sand drains, no satisfactory reliable laboratory test method has been devised for measuring permeability as it affects radial flow. Determination of field permeabilities may provide a more reliable method of estimating rates of consolidation.

The field permeability test is made by installing, in the soil layer being studied, a porous stone about 1-1/2 inches in diameter and one to five feet in length, with a plastic tube leading from the porous stone to the ground surface. The porous stone is commonly installed in a 2-1/2 inch diameter hole, with sand backfill. A schematic drawing of the permeameter is shown on Figure 11. The piezometer is left in place for several days before taking readings, to allow the system to reach pressure equilibrium. After careful checking to assure that the piezometer is functioning properly, the piezometer head in the tubing is lowered by pumping out water. The piezometer level is measured and recorded at measured time intervals, and the ratio of measured head to original head is computed for each time interval; the log of this ratio is plotted against time. The basic time lag and coefficient of permeability are then computed, using the method described in Waterways Experiment Station Bulletin No. 36.(5)

Instead of lowering the piezometer level and performing the test with a rising level, as outlined above, the piezometer level may be raised by adding water to the system, and the test performed with a falling head. The question of which method may be more advantageous depends on conditions. Work done by this department, based on the procedure described by Waterways Experiment Station, indicates better correlation between field permeabilities and observed rates of settlement than by the use of coefficient of consolidation determined in conventional laboratory consolidation tests. However, much additional investigation will be necessary before any conclusions of general validity can be formulated.

CONSOLIDATION OF PEAT

It is the author's opinion that consolidation properties of fibrous peats cannot be evaluated accurately by application of the usual consolidation testing procedures and the generally accepted theories of consolidation. For inorganic clays there is reasonably good agreement between estimated and observed amount of settlement, and at least a semblance of correlation between computed and actual rates of settlement when based on consolidation tests of good quality undisturbed samples. This does not appear to be true in the case of fibrous peats, where numerous difficulties are encountered in the interpretation and application of consolidation test data, and the accuracy of settlement predictions for peats is likely to be of a low order.

There is need for a rational system of classifying organic soils and peats. Such soil may consist of pure peat with virtually no disintegration of the plant forms, various gradations of mineral grains and partially decomposed plant remains, or soil composed primarily of silt and clay but with high organic content. Figure 12 is an enlarged photograph of a sample of peat from the Delta area north of the Antioch Bridge. In its natural state the moisture content of the samples was from 550% to 650%. In order to show more clearly the texture of the peat, a cross section of an oven dry sample 1 inch high by 2 inch diameter was photographed and enlarged three diameters. Also shown is an enlarged photograph of a similar sample after consolidating under a load of 8 tons per sq. ft., and oven drying. Note the dense texture and apparent absence of fibers in the consolidated specimen.

The shapes of time-consolidation curves for peats vary greatly, and it is difficult to select a typical curve, especially at loads of 1/2 ton per sq. ft. or less. Figure 13 illustrates the more common types of time-consolidation curves, and Figure 14 a load consolidation curve for peats in the Delta area of California with moisture contents of over 1000%. Work done by this department confirms the conclusions reached by Thompson and Palmer(6) and others, that there is little similarity between time-consolidation curves for peat and those obtained for clays. The concept of primary and secondary consolidation, generally accepted for clay soils, is not considered applicable in the case of peats, where rate of consolidation does not appear to be a function of drainage. This is evidenced by the fact that the rate of strain in the consolidation tests of peat is independent of the height of specimen.

In general, the estimates of settlement derived from consolidation tests of peat with the usual twenty-four hour loading period are considerably lower than actual settlements recorded during and after construction. It has been found by Mr. W. A. Lewis(7) that after a load increment in the consolidation test is left in place for fifty days the rate of consolidation has become exceedingly small, although slight compression may still be occurring. This long loading period was not proposed as a routine test procedure, and would not be practicable for such use. Some modified method of testing and interpreting the test data is needed which will permit more realistic estimates of settlement of fills constructed over peat beds.

If it is accepted that the rate of consolidation of peat is not a function of drainage, it is logical to question the use of vertical sand drains in peat deposits. There is substantial, although not conclusive evidence, that sand drains effect an increase in shear strength of the peat during construction. Unfortunately, no records are available on embankments of critical height constructed by California Division of Highways over identical peat deposits with and without sand drains. Only by such evidence could it be proved conclusively that the use of sand drains permitted construction of a stable embankment which, without sand drains, would have resulted in shear failures. However, based on California experience with construction over peat beds, it is believed that vertical sand drains accelerate the increase in strength of peat deposits during the loading period. On the Antioch project referred to above, for example, a total thickness of embankment as great as 25 ft. was constructed with sand drains, with no measurable displacement or shearing, even though settlements of as much as 18 feet have occurred. On previous fill construction over similar peat beds in the same region, shear failures or displacement of the peat occurred under much lighter loading. At this sand drain location the moisture content of the peat before construction averaged about 600%, and unconfined compressive strength ranged from 0.2 to 0.3 tons per sq. ft.; samples taken from the same location after completion of the embankment had moisture contents of about 300% and unconfined compressive strengths of 0.4 to 0.8 tons per sq. ft.

It has been suspected for several years that settlement analyses of peat by the conventional methods developed for clays are not always reliable. Because no better method has been developed, the magnitude and rate of settlement have been estimated by the same procedures as for clay soils. Previous experience in construction over the peat deposits of the delta area of the California central valley has been helpful in estimating settlements on proposed construction in the same region, but peat soils in other areas may not have the same consolidation or strength characteristics.

In general, the settlement occurring during construction approximates the ultimate settlement for the peat as computed from the 24-hour consolidation tests by the same procedure used for clays, but there are numerous unexplained discrepancies. The rate of settlement greatly diminishes a short time after loading of the fill is discontinued; thereafter, the settlement follows a straight line on a semilog plot, but the slope of the line varies markedly even between points where the fill loads are comparable and the peat appears similar as to thickness of bed and moisture content. For light fills over these peat deposits the post-construction rate of settlement commonly ranges between 0.5 and 1.5 ft. per log cycle of time, and the rate does not appear to be proportional to the thickness of the peat layer.

Comparisons of computed and observed settlements of embankments over peat deposits are shown by Figures 15 to 19. Two of the comparisons, Figure 15 and Figure 18, are for areas in which sand drains were installed. The settlement calculations for all of the theoretical curves were based on conventional consolidation tests with a 24-hour loading period for each load increment. The time-settlement relationships for the peats, however, were derived by use of field permeabilities.

Note that in the sand drain area at Station 11, Figure 18, the total settlement to date has been 16.2 feet, but only about 0.6 ft. occurred after construction. The calculated ultimate settlement was 15.0 ft., or 93% of the measured settlement. In this instance there was good agreement between estimated and actual settlements.

At the other sand drain area at Station 80 on the Antioch project, Figure 15, the measured settlement to date has been 12.3 ft., which is 86% of the calculated ultimate, and the settlement is not completed. In this case, however, almost four feet of settlement has taken place subsequent to paving, and the fill is still settling.

At the other three locations, where no sand drains were installed, the settlements were as follows:

Figure	Station	Calculated Ultimate	Observed To Date	After Paving
16	113	1.6 ft.	2.0 ft.	1.0 ft.
17	214	3.8 ft.	4.6 ft.	0.6 ft.
19	121	4.2 ft.	4.5 ft.	2.5 ft.

It is evident that there is fair agreement between calculated and observed total settlements, but the magnitude of the post-construction settlement varies widely.

It is questioned whether primary consolidation can be identified in the consolidation tests of peat; if there is primary consolidation it probably takes place during the very early stage of the test, and is complete in one to ten minutes. The long-time consolidation, which is evident in both the laboratory test and under field loading, is proportional to the logarithm of time. This phenomenon is not clearly understood, but the long-term compression may be due to plastic deformation, slow structural failure of the peat fibers, and slow disintegration of the plant forms in the past.

It is possible, also, that this is not entirely secondary consolidation, at least during the intermediate stages of the total consolidation period. The initial permeability of the peat before loading is relatively high, but diminishes rapidly as the peat is compressed; accordingly, at some time subsequent to the completion of the initial "primary consolidation" the permeability of the peat may decrease to the point where the rate of consolidation is influenced or controlled by the dissipation of pore pressure. For example, on the Antioch project mentioned above, field permeabilities of the peat layer were determined before and after loading at three locations. The field permeabilities were as follows:

Permeability in Ft./Hr.

Station	Initial	During Loading	Settlement At Time Of Second Measurement
70+75	1.2×10^{-3}	1.2×10^{-4}	4.0 ft.
214+00	1.4×10^{-2}	2.6×10^{-5}	3.6 ft.
215+00	1.7×10^{-2}	3.3×10^{-5}	4.4 ft.

On other projects in the Delta area the permeabilities of the peat, as measured by the field method, ranged from 1×10^{-2} ft./hr. for no loading to 1×10^{-5} ft./hr. under fills of ten to fifteen feet. The supporting data are admittedly meager, and these results are cited as being perhaps indicative, rather than conclusive.

Fairly adequate test data and long-time settlement records are available on several California highways projects involving embankment construction over peat beds. It was hoped that by analysis of these data a reliable empirical or rational method could be formulated for estimating rate and magnitude of settlement. However, because of numerous unexplained anomalies and discrepancies, no satisfactory procedure has yet been developed for predicting settlement of embankments constructed over thick beds of peat.

SUMMARY AND CONCLUSIONS

Various methods of embankment construction over marsh deposits have been employed on California highways, and each has been successful when used where conditions were appropriate. By proper utilization of the principles of soil mechanics the performance of the structure can be predicted for the various alternate construction methods. The economic phases of the design cannot be ignored, and a rational decision on type of treatment can be reached only by thorough evaluation of engineering economics, with consideration of cost of construction, probable costs of maintenance and reconstruction, and the adequacy of service to traffic.

It is seldom economical to design embankments across marsh deposits without some type of special treatment unless preliminary investigation indicates that the foundation soil will support the proposed loading with some reasonable factor of safety. The time element is often the controlling factor -- stable embankments and levees have been successfully constructed over marsh deposits by slow stages during a period of several years, whereas, attempts to build similar embankments in one or two years have resulted in complete failure.

Where less important roads carrying light traffic must traverse extensive marsh deposits it may be possible to support light embankments without stripping of the marsh soil or other costly treatment. Even though settlement of the road may occur for many years, and periodic restoration to grade may be required, this may be less costly than extensive foundation treatment; or, even if the over-all ultimate costs of the two were comparable, funds would not be available for the large initial investment involved in the stripping or sand drain treatment.

The most positive treatment of marsh deposits is, of course, removal of all soft compressible material and replacement with granular fill material. If the depth of the unstable soil is only a few feet, stripping is generally the most economical solution but is not feasible where the peat or mud is of great depth. Vertical sand drains have been used successfully in cases where the rate of consolidation of the marsh soil was sufficiently rapid that the weak material acquired early shear strength to prevent shear failures, and where most of the primary settlement occurred during the construction period. The sand drains are more likely to be effective if the fill load can be applied slowly over a long period of time, and a surcharge can be left in place for several months before the road is paved.

No one method of construction over marsh deposits is suitable for all conditions. A rational design can be achieved only after thorough investigation and analysis by judicious application of the principles of soil mechanics, together with an engineering economics study.

ACKNOWLEDGMENTS

All of the foregoing data were selected from job records in the Materials and Research Department of the California Division of Highways, which is under the supervision of F. N. Hveem, Materials and Research Engineer. The author wishes to acknowledge, with sincere appreciation, the assistance given by all those who helped compile the data. Special credit is due Travis W. Smith for his valuable suggestions and assistance, and to William G. Weber and William F. Kleiman who helped to compile and analyze the data.

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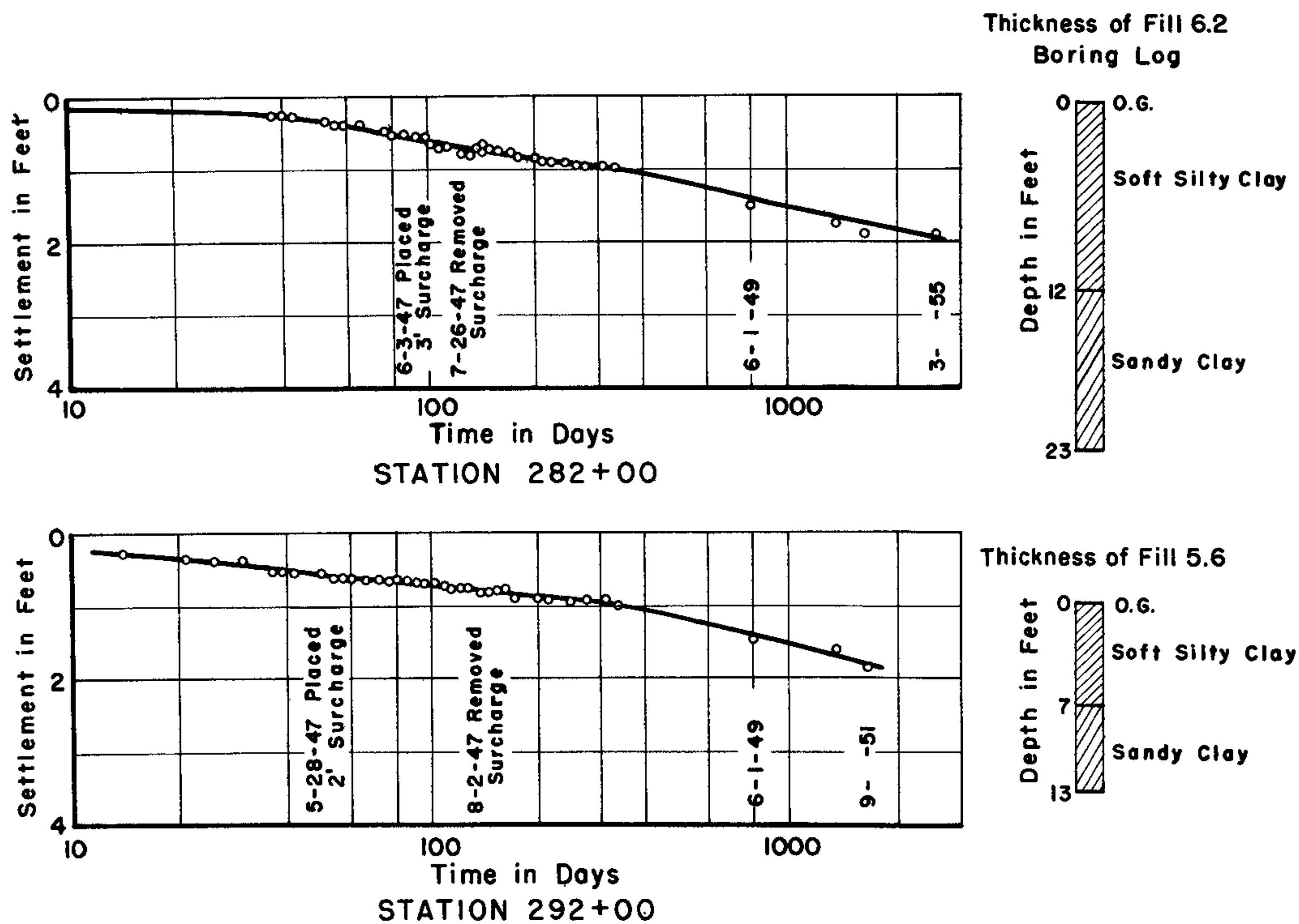


FIG. I OBSERVED SETTLEMENT
BAYSHORE FREEWAY

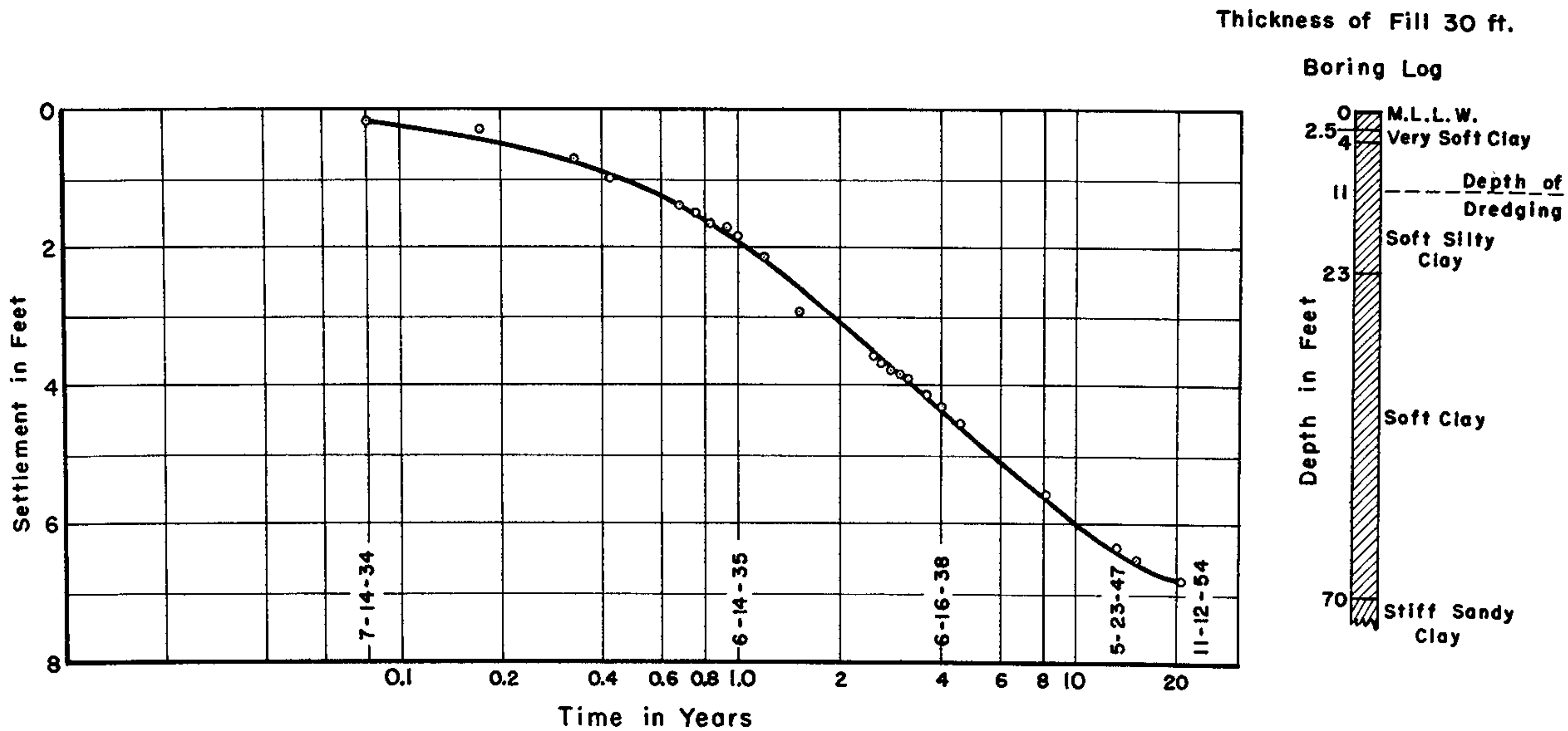


FIG. 2 OBSERVED SETTLEMENT

San Francisco Oakland Bay Bridge Approach

STA. 317 + 50 (L 21-S)

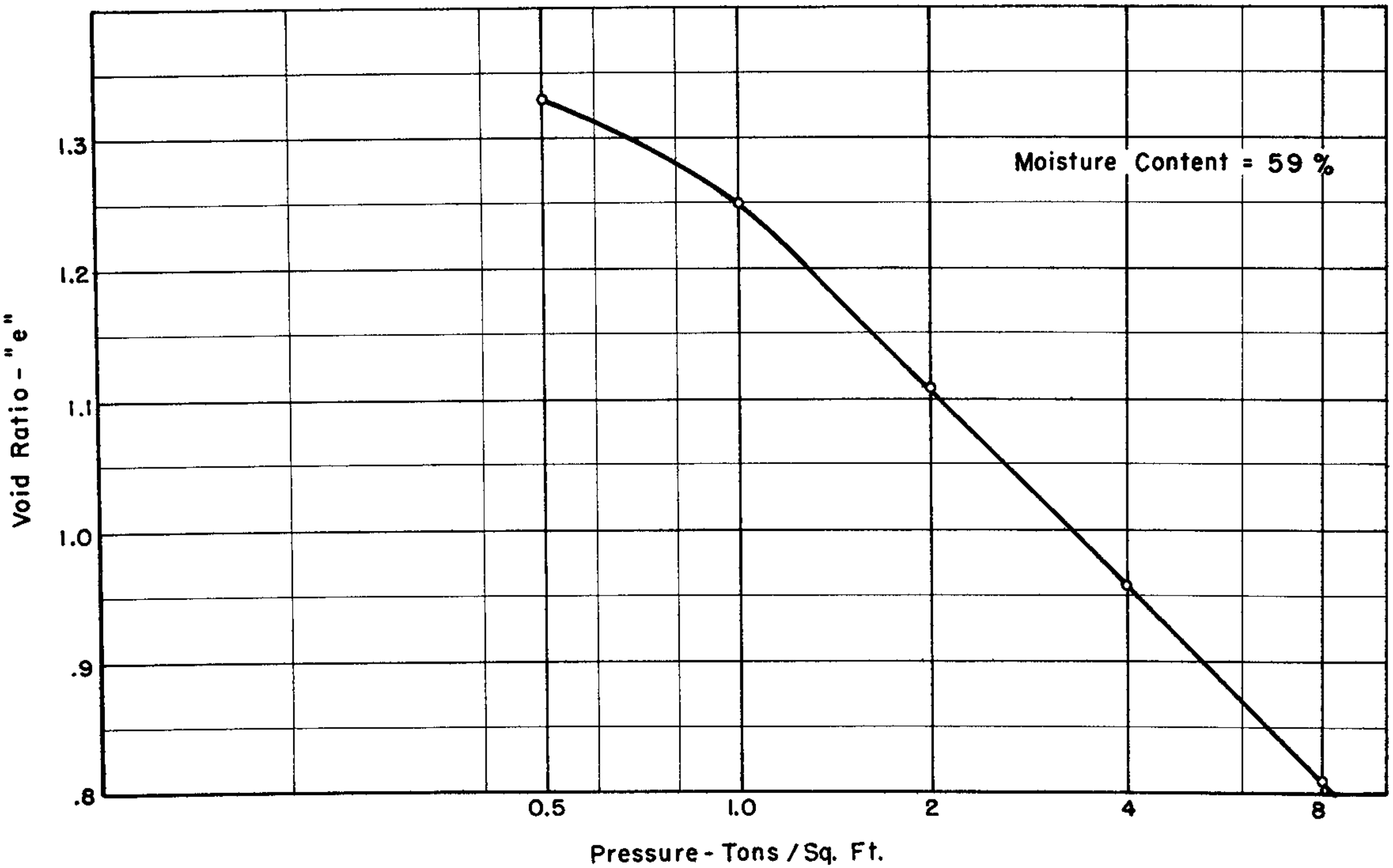


FIG. 3 TYPICAL PRESSURE - VOID RATIO CURVE, SOFT SILTY CLAY
San Francisco - Oakland Bay Bridge - East Approach

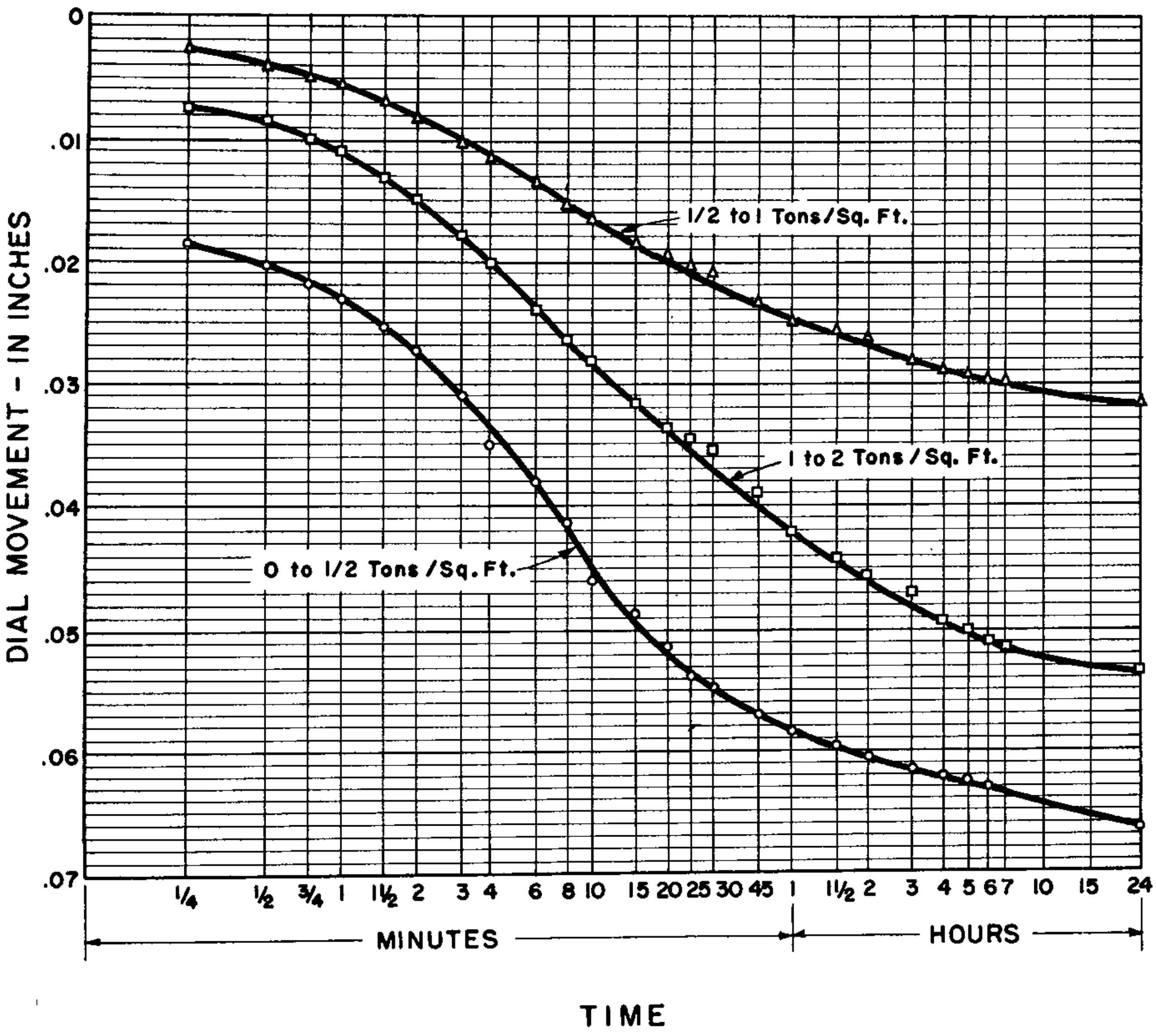


FIG.- 4

TYPICAL LABORATORY TIME-CONSOLIDATION CURVES

SOFT SILTY CLAY

San Francisco-Oakland Bay Bridge, East Approach

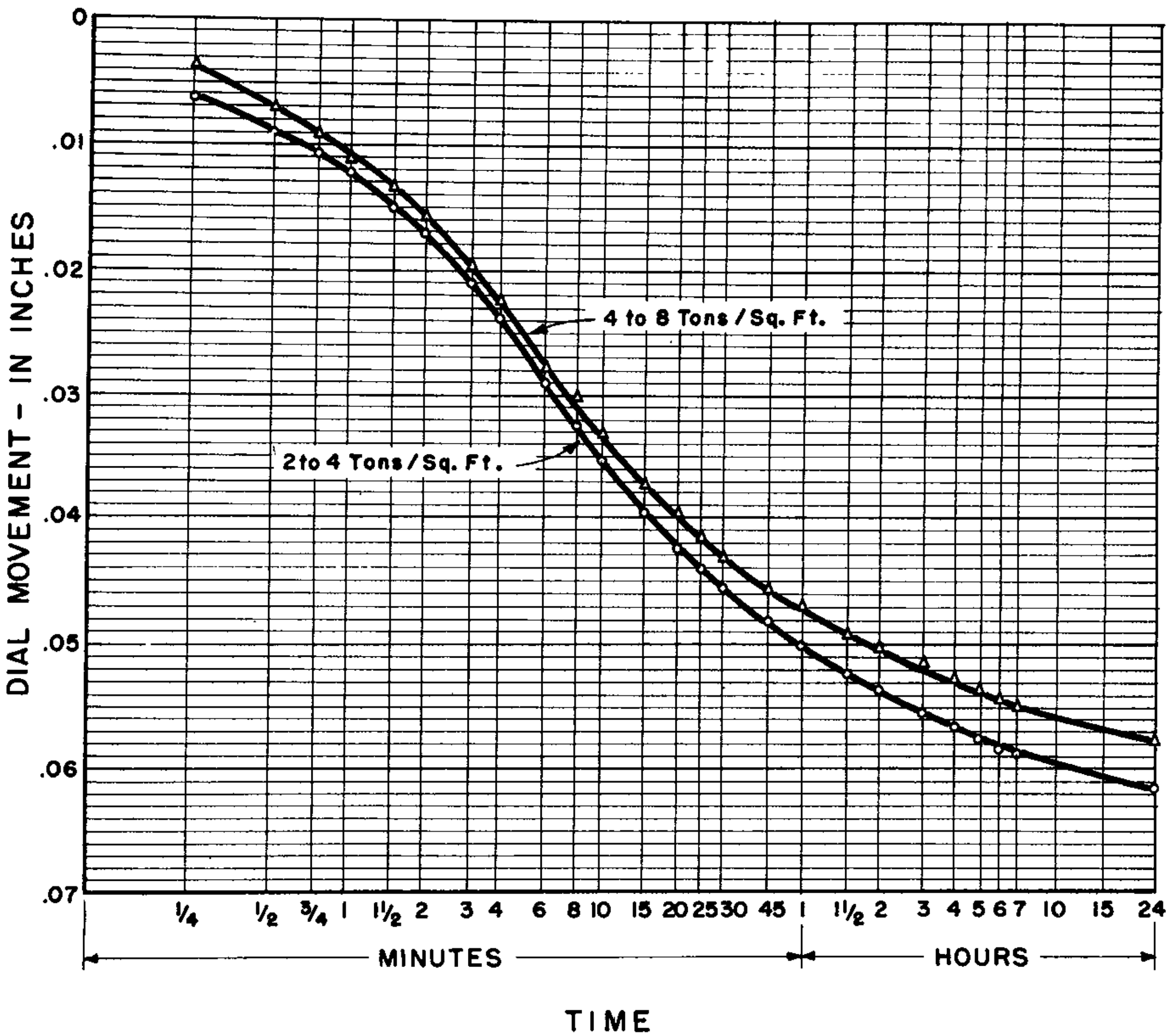


FIG.- 5

TYPICAL LABORATORY TIME-CONSOLIDATION CURVES

SOFT SILTY CLAY

San Francisco - Oakland Bay Bridge, East Approach

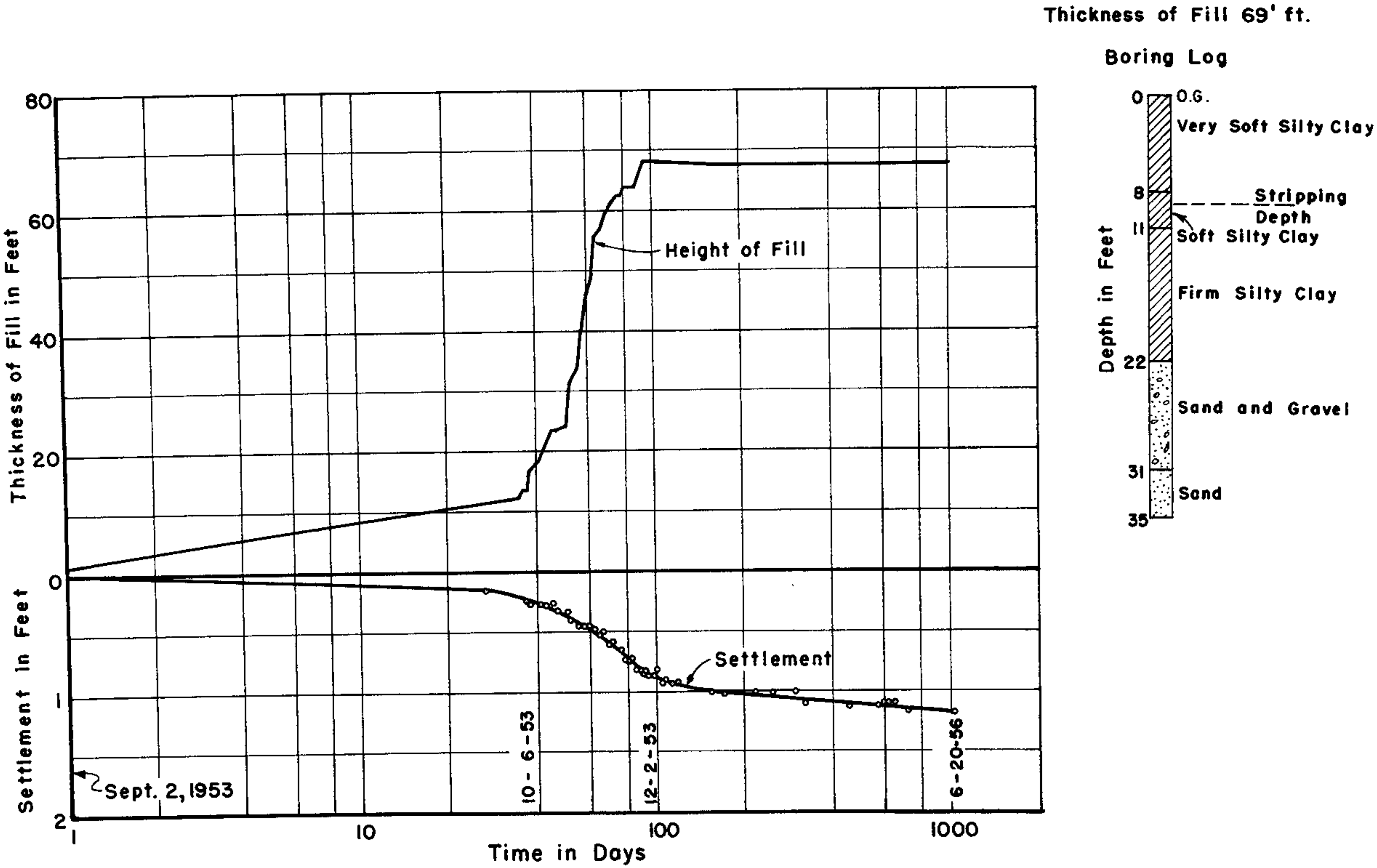


FIG. 6 OBSERVED SETTLEMENT
Near Petaluma
IV- Son-I-F STA. 857+60

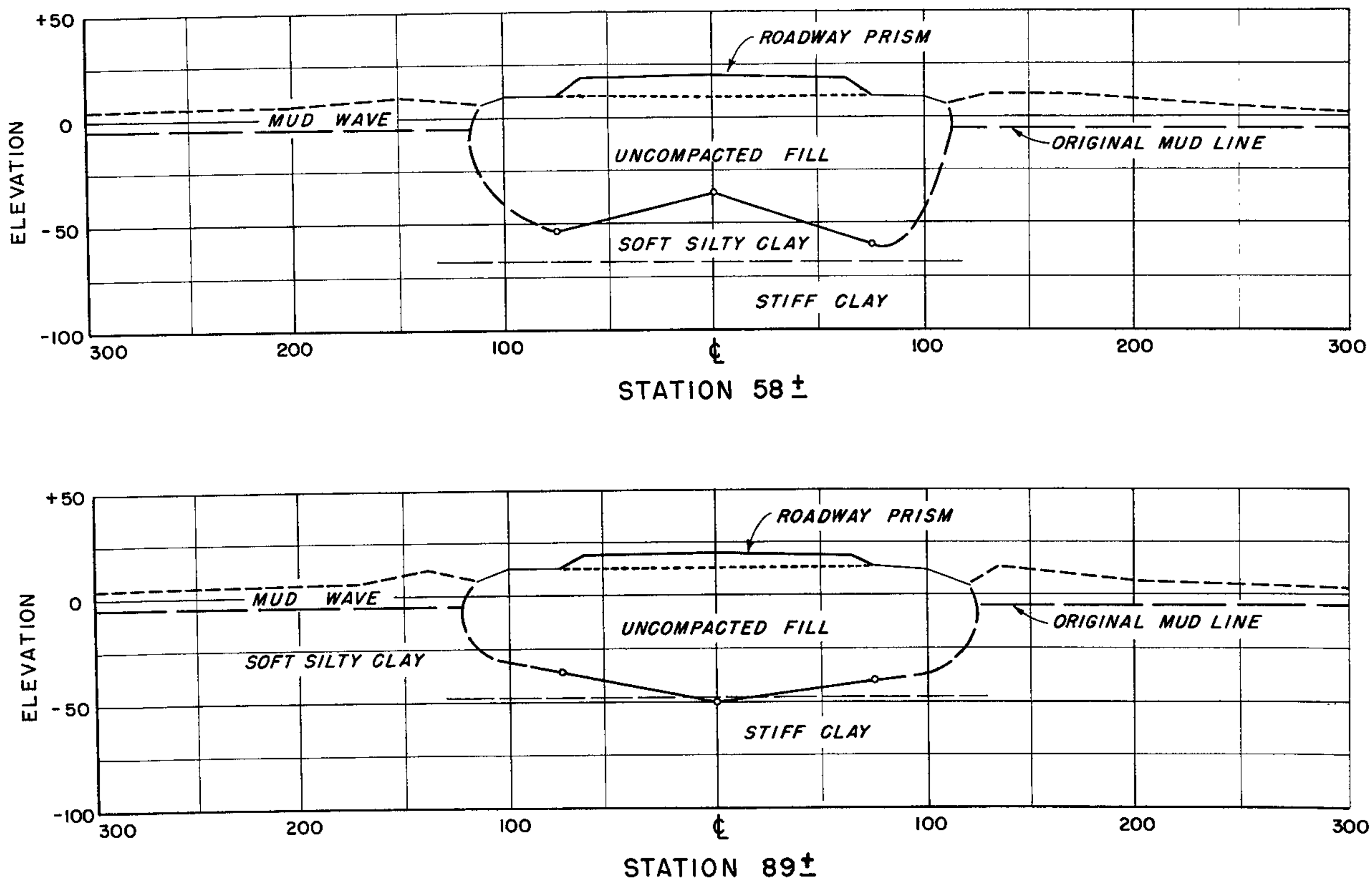
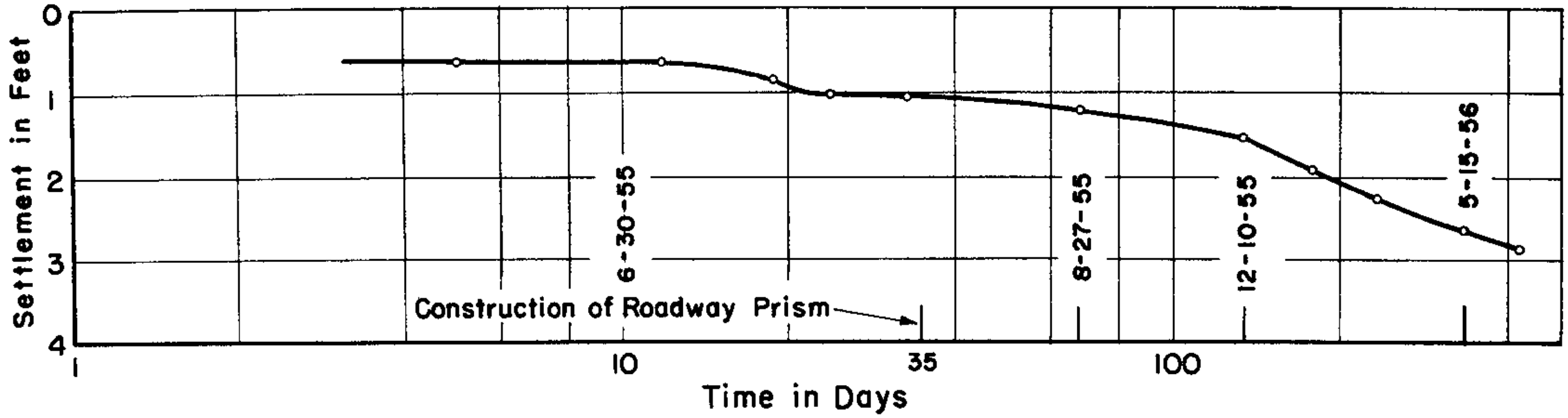
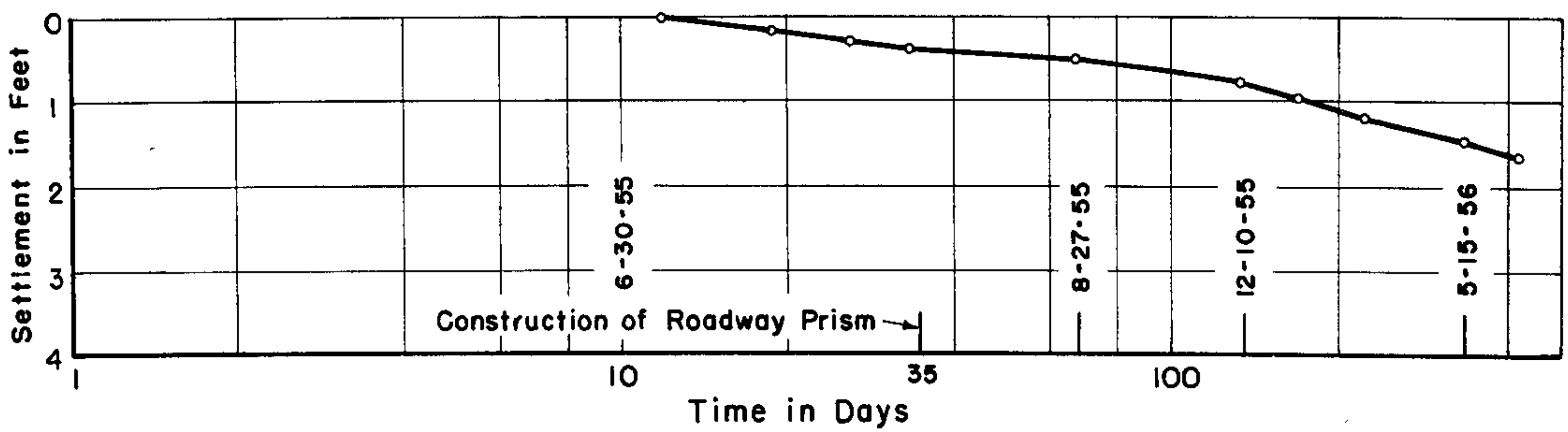


FIG.-7 CROSS-SECTIONS OF FILL AT CANDLESTICK COVE



STATION 58+00



STATION 89+00

FIG.8 OBSERVED SETTLEMENT
CANDLESTICK COVE

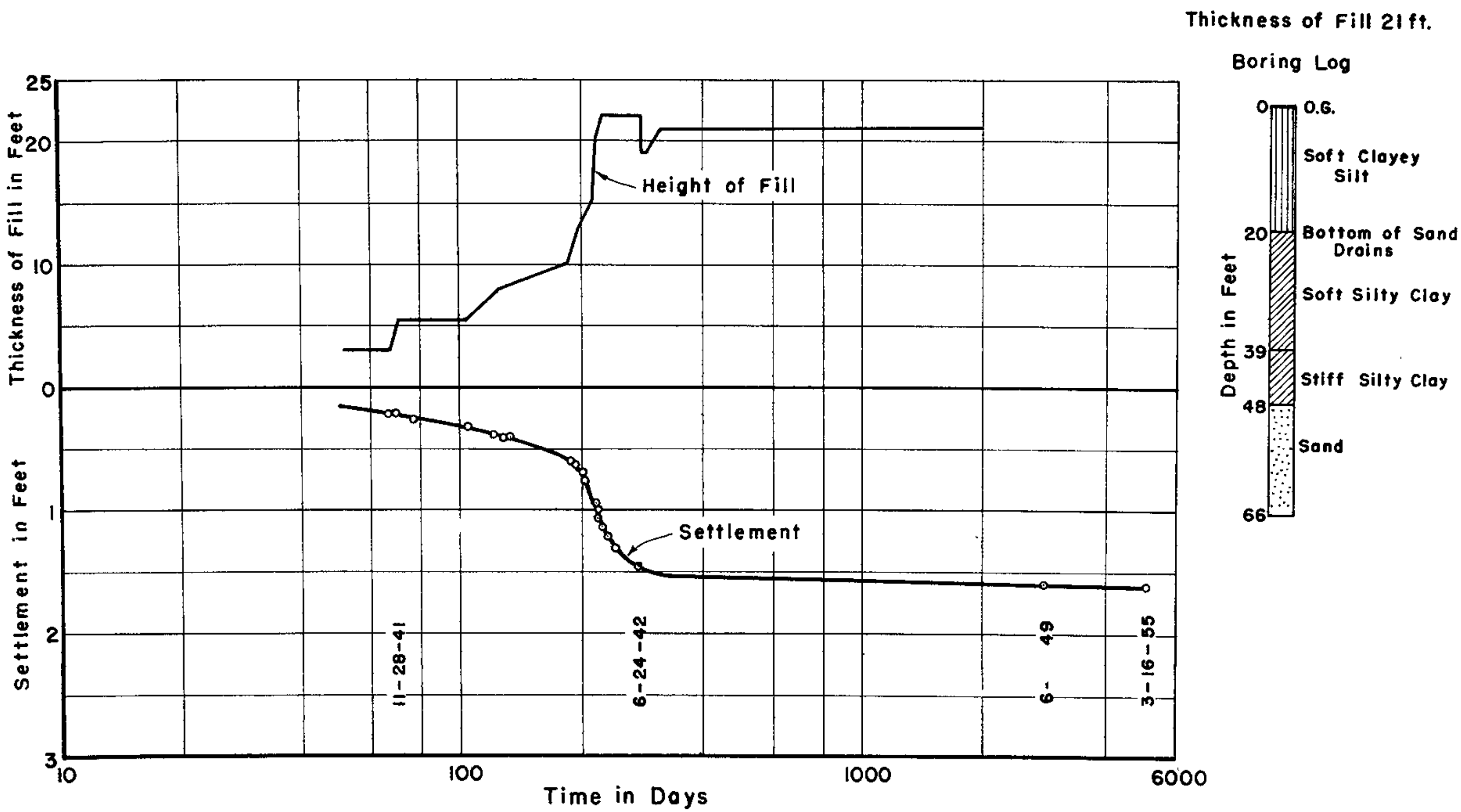


FIG. 9 OBSERVED SETTLEMENT
Eureka Slough-Sand Drain Area

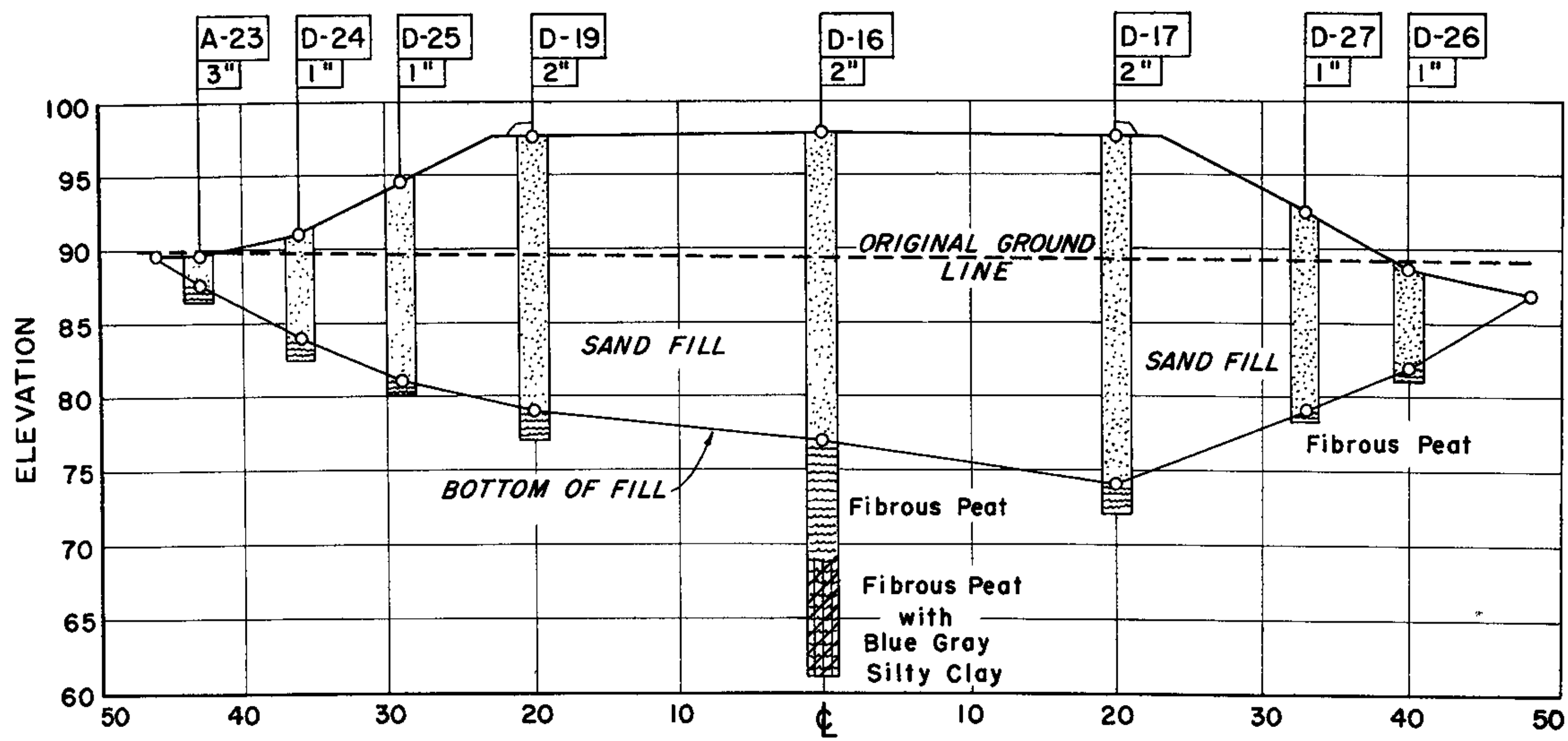


FIG. 10

CROSS-SECTION OF FILL IN SAND DRAIN AREA

North Approach Antioch Bridge

X - Sac. - 11 - C

Station 18 + 75

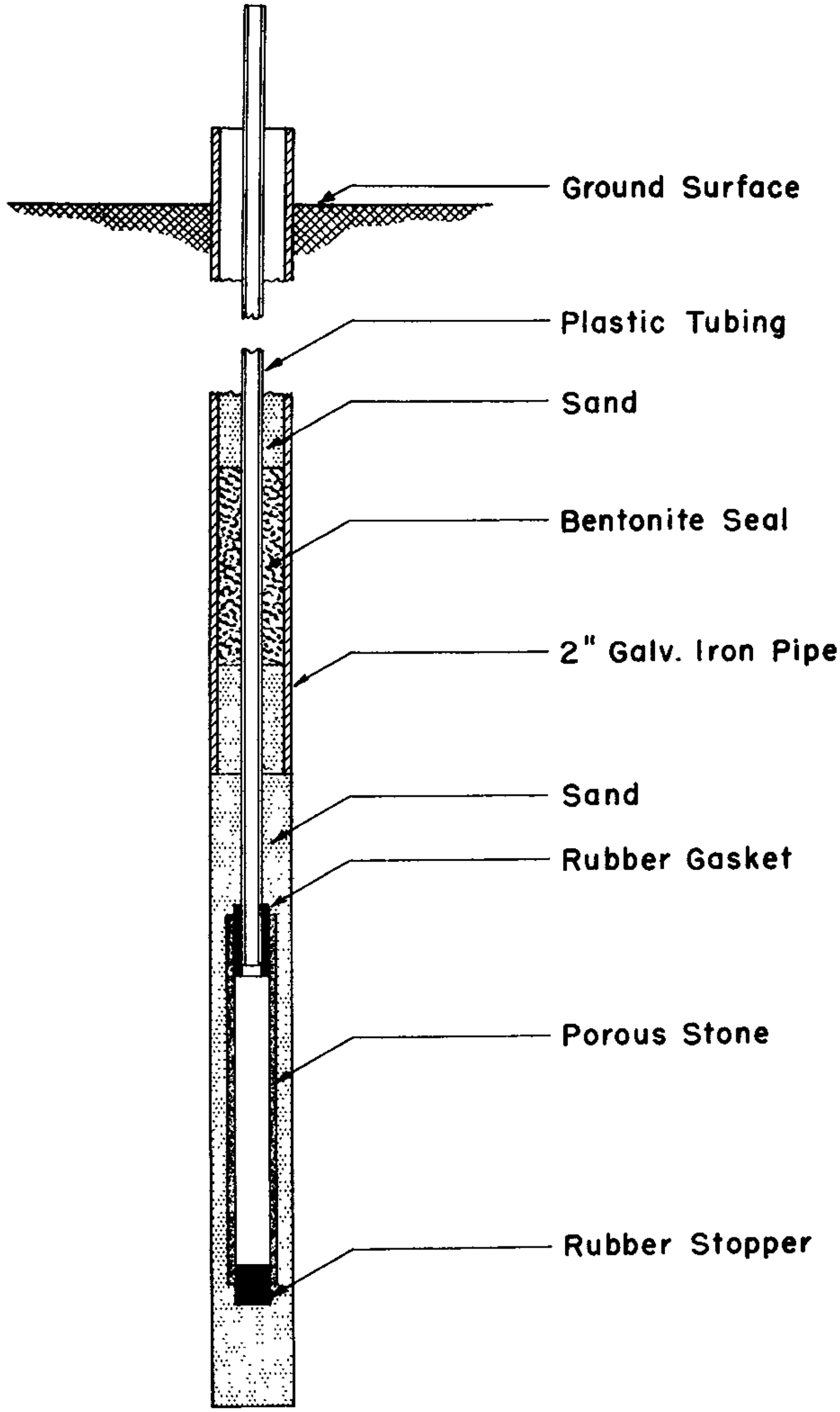


FIG. 11 SCHEMATIC SKETCH OF PIEZOMETER FOR FIELD PERMEABILITY MEASUREMENT

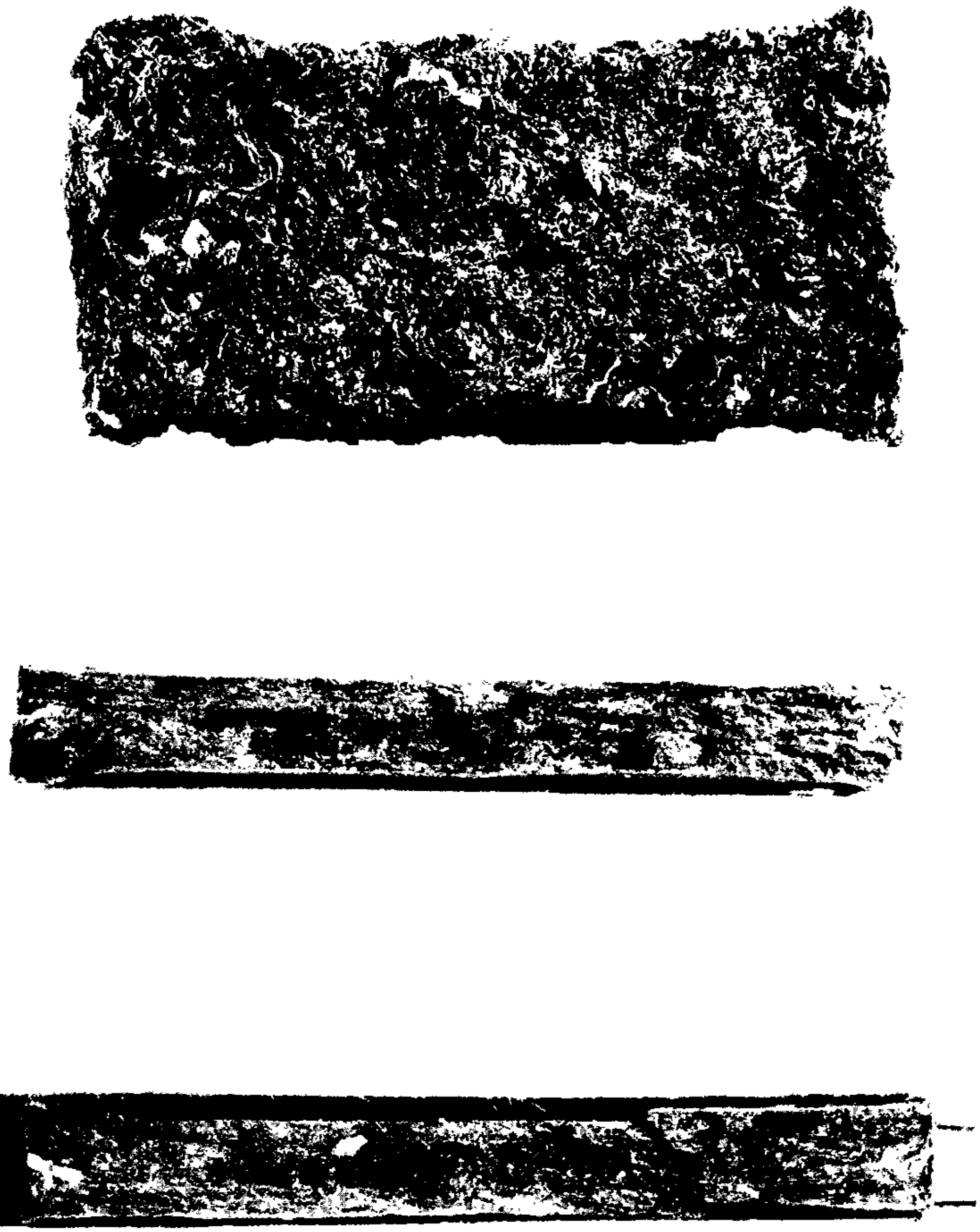


Fig. 12 **Oven-dried peat.** Original water content of 600% Upper photo shows vertical section of unconsolidated material. Lower photo shows similar material after 75% consolidation under 8 T.S.F. load.

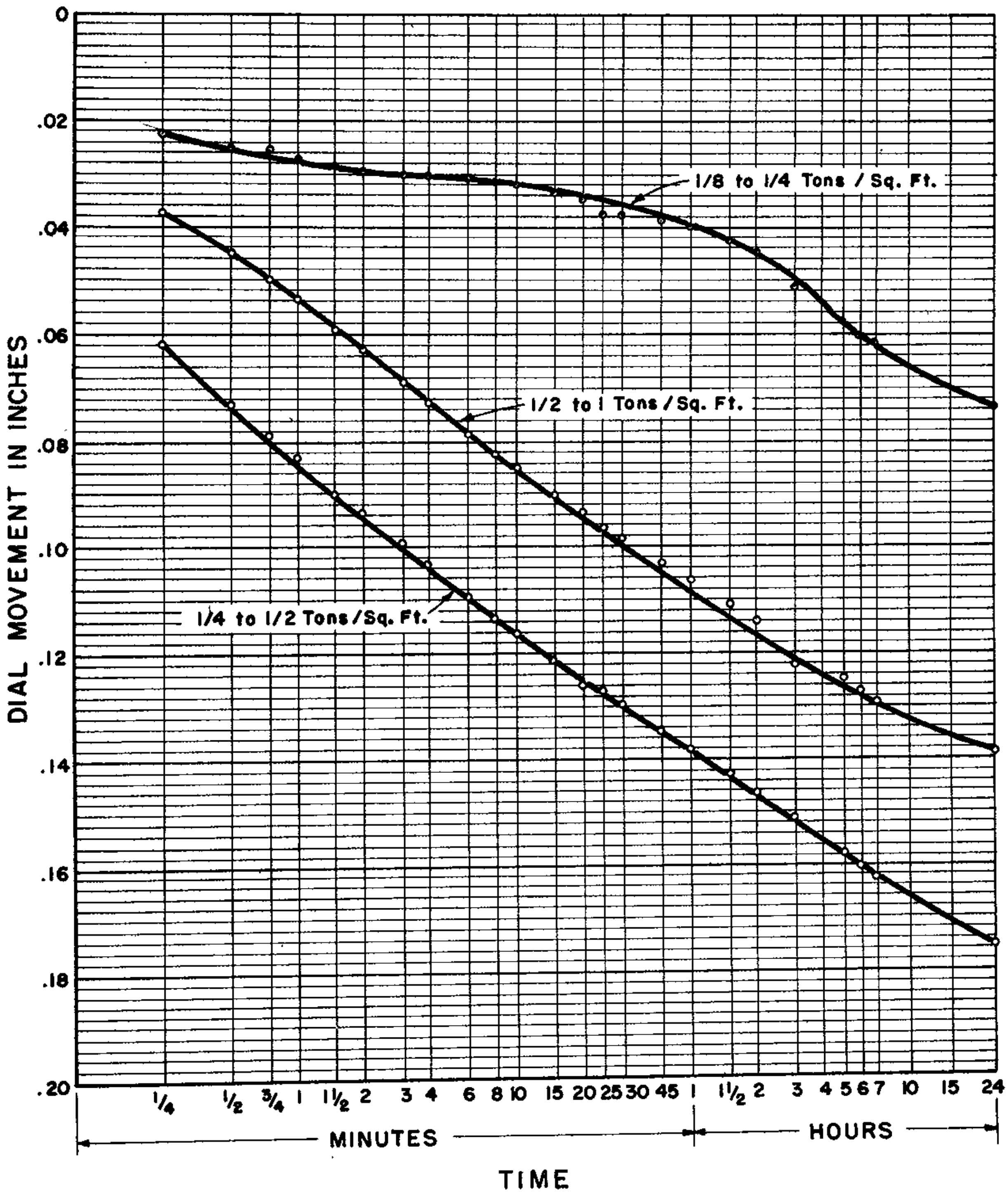


FIG. 13

TYPICAL LABORATORY TIME - CONSOLIDATION CURVES

Antioch Peat

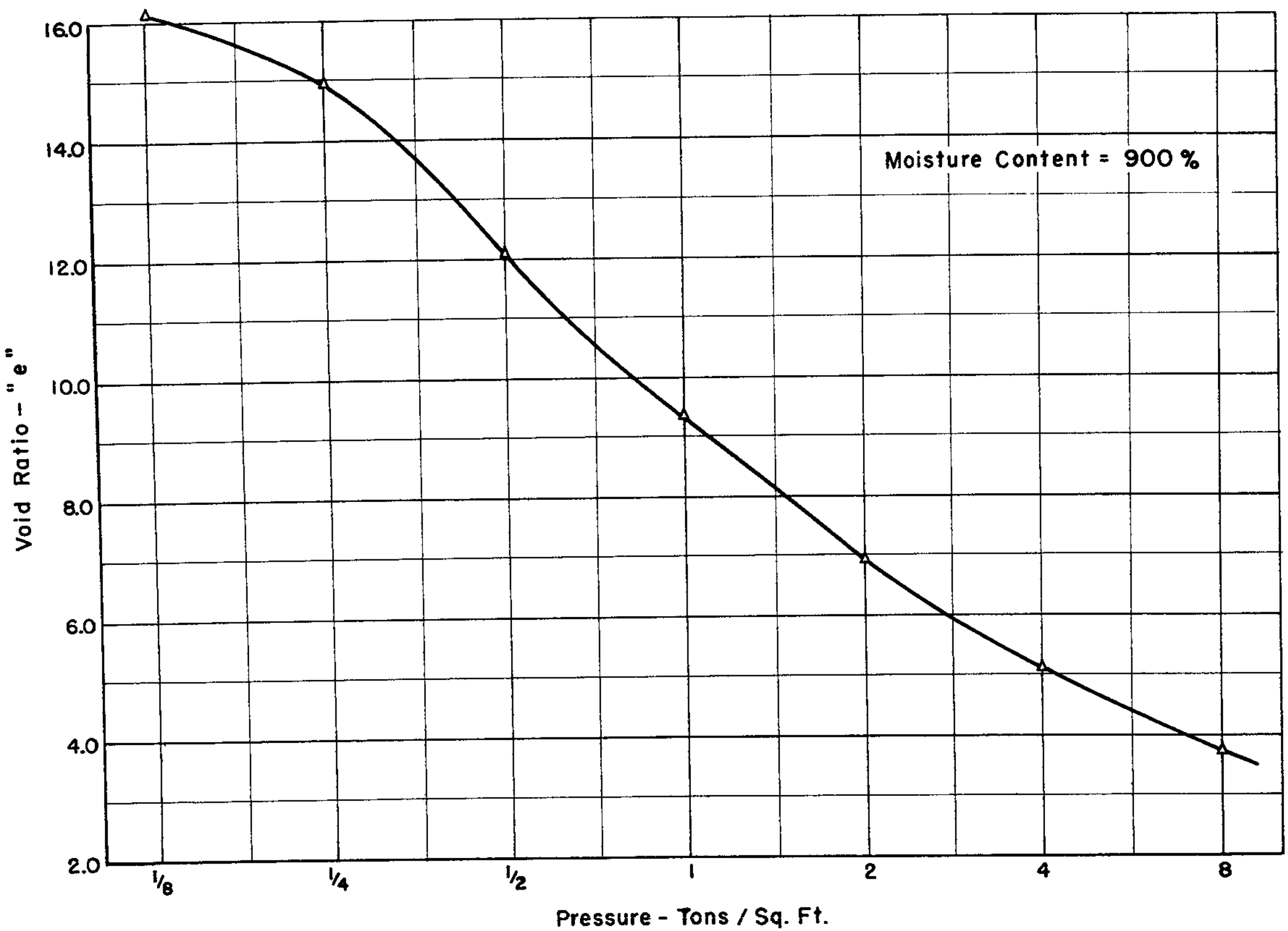


FIG. 14 TYPICAL PRESSURE - VOID RATIO CURVE, PEAT
Mokelumne River to Potato Slough

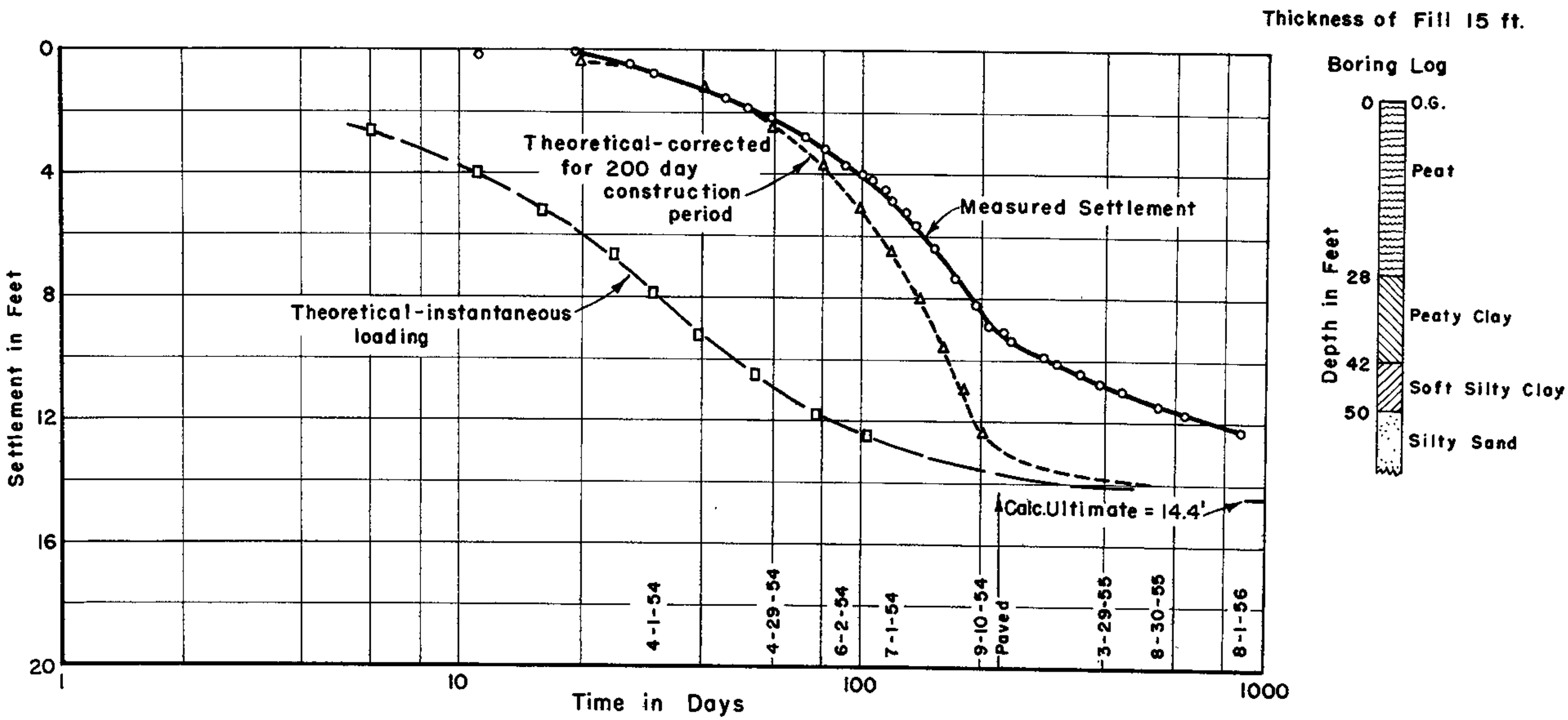


FIG. 15 COMPARISON OF MEASURED & THEORETICAL SETTLEMENT
North Approach to Antioch Bridge-Sand Drain Area
STA. 80 + 10

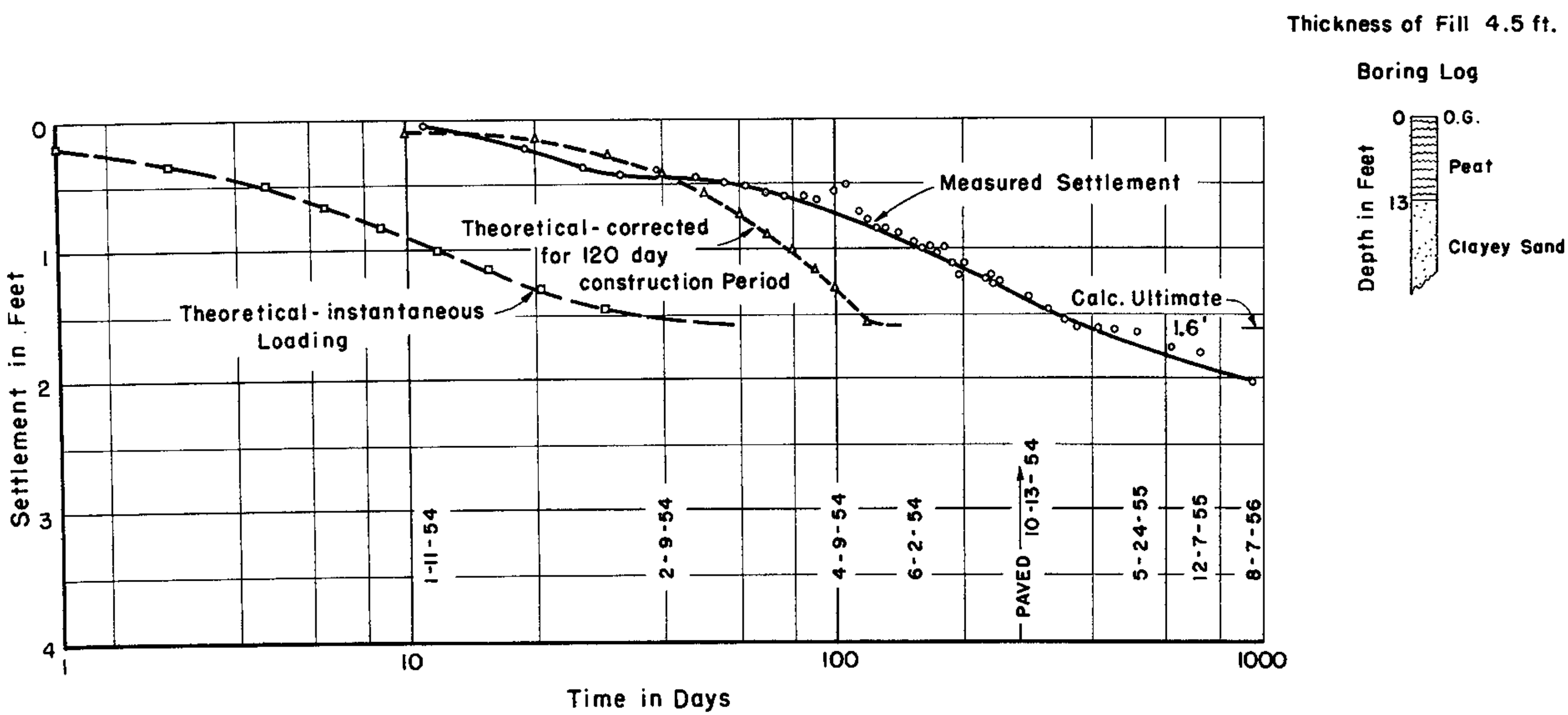


FIG. 16 COMPARISON OF MEASURED & THEORETICAL SETTLEMENT
North Approach to Antioch Bridge
STA. 113+00

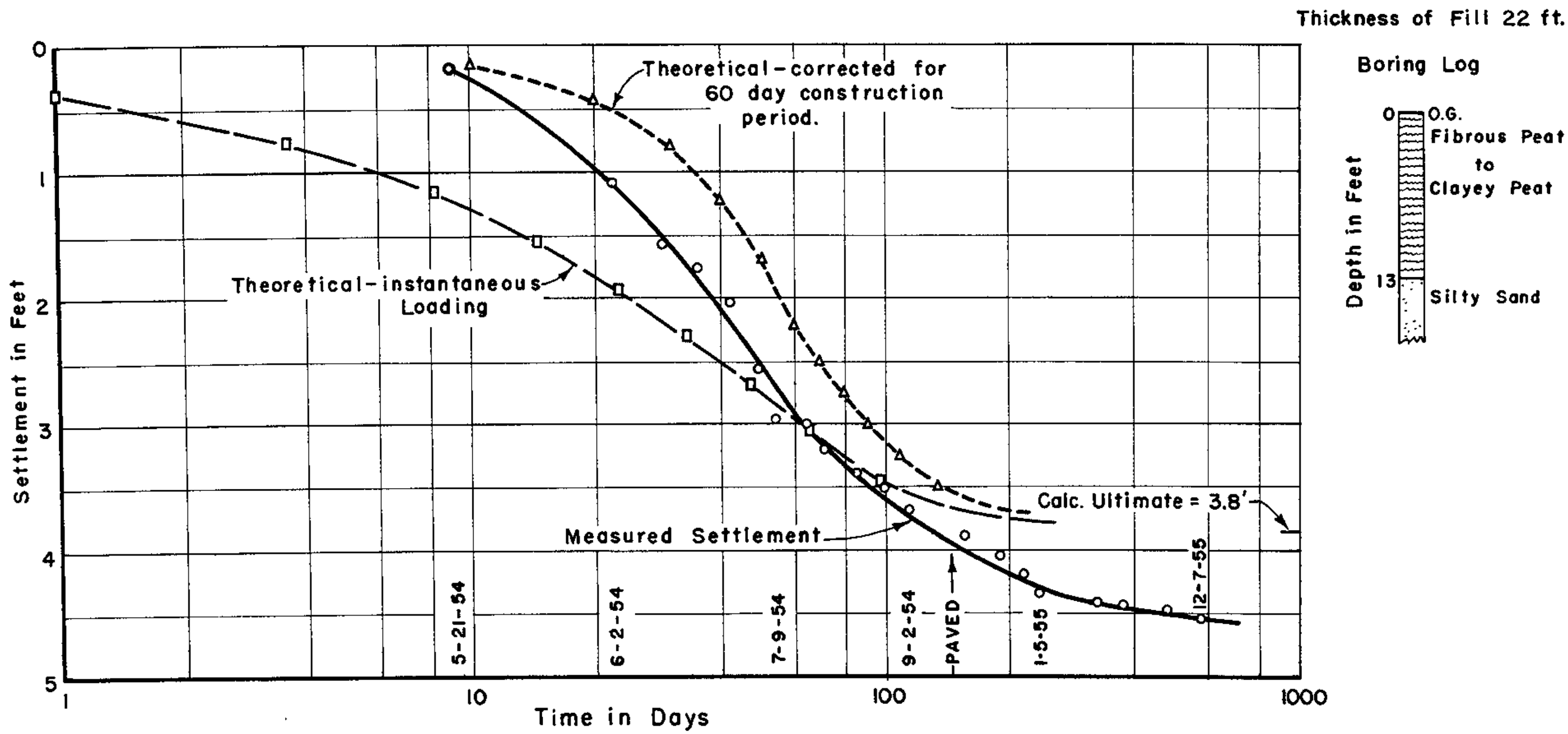


FIG. 17 COMPARISON OF MEASURED & THEORETICAL SETTLEMENT
North Approach to Antioch Bridge
STA. 214 ±

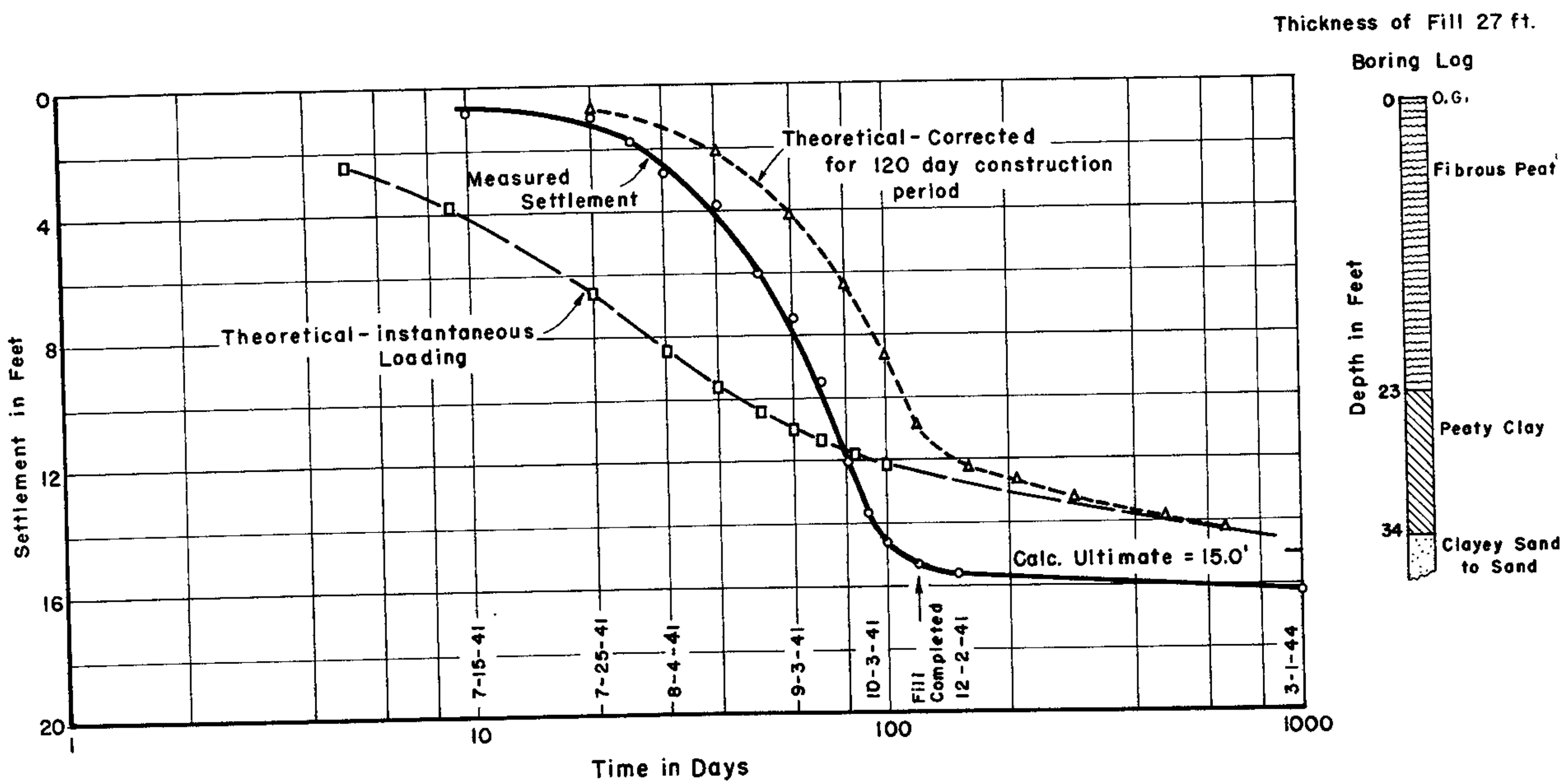


FIG. 18 COMPARISON OF MEASURED & THEORETICAL SETTLEMENT
Mokelumne River to Potato Slough - Sand Drain Area
STA. 11 ±

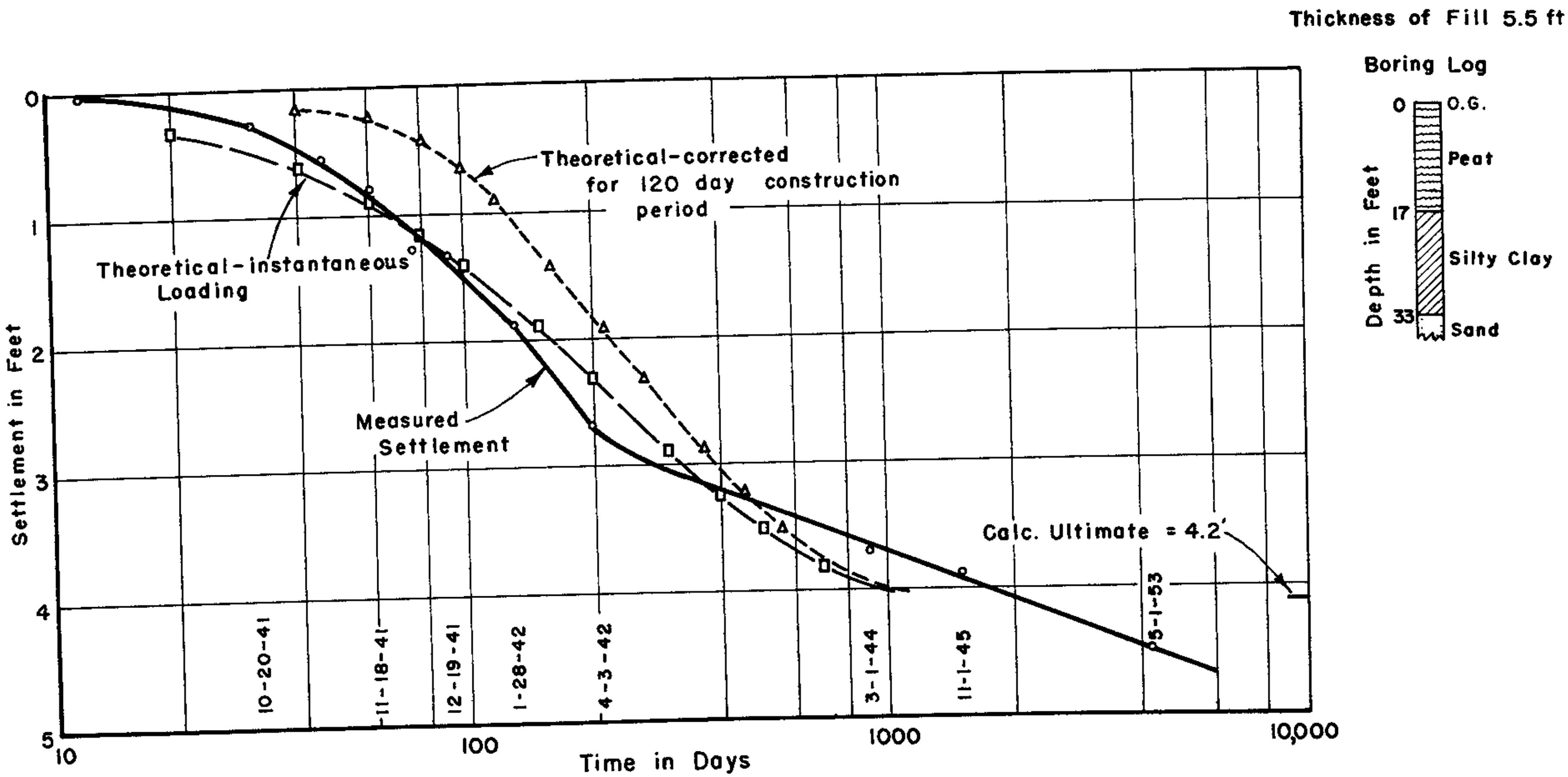


FIG. 19 COMPARISON OF MEASURED & THEORETICAL SETTLEMENT
Mokelumne River to Potato Slough
STA. 121 ±